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DESIGN FEATURES OF BEAR RIVER HYDROELECTRIC PROJECT

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POWER DIVISION

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DESIGN FEATURES OF BEAR RIVER HYDROELECTRIC PROJECT

J. Barry Cooke,[1] A. M. ASCE

INTRODUCTION

The P.G. & E. expansion program has included some 10 hydro electric projects in recent years. They cover a wide range in flows, heads and other features. However, the recently completed Bear River Project is considered of particular interest and this paper will review the design features of the Bear River Project.

This Bear River, one of many, is a tributary to the Mokelumne River and is located some 120 miles due east of San Francisco. A two and one-half mile power tunnel through a ridge separating the two rivers develops 2110 feet of head. The 40,000 HP development includes a 230 ft. high rock fill dam, a combination surge tank and creek diversion, a 2110 ft. head penstock and a multiple jet vertical shaft impulse turbine. The general plan, Fig. 1, shows the project layout and structures. The design features will be reviewed beginning at the storage dams and following the flow down to and through the power house.

Mokelumne River Development

The Bear River Project is not just another plant but rather an integrated part of the Mokelumne River Development and it seems appropriate to briefly review the overall plan. Between 1900 and 1932 the Mokelumne Power Development consisted of small high altitude rock-fill storage dams, wooden flume and canal conduit and the 25,000 HP old Electra Power Plant. In 1930 a plan for the full development of the river was conceived and the recent completion of Bear River Project completes the third and last phase of that plan, bringing the total capacity up to 277,000 HP of valuable peaking power. The Bear River Project became economic only on completion of the downstream plants which, in benefiting from this storage, contribute to paying for the dam. The following tabulation summarizes the development:

Phase	Completion Date	Plant	Head-Ft.	Peak Flow-cfs	HP
1st	1932	Salt Springs	255	650	16,000
1st	1932	Tiger Creek	1219	625	78,000
2nd	1948	West Point	312	650	20,000
2nd	1948	Electra	1268	1000	123,000
3rd	1952	Bear River	2110	210	40,000
					277,000

1. Senior Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

On the upper Mokelumne are four reservoirs which catch the snow melt at an elevation of 8,000 feet. Farther down the river at elevation 4000 is the large Salt Springs reservoir created by the 328 ft. high Salt Springs rock fill dam. Upstream from the new Bear River Reservoir is an existing smaller rock fill dam forming Upper Bear River Reservoir. These seven reservoirs provide the storage and farther downstream four forebay reservoirs give complete "water saving" flexibility of power operation. In passing through the five power plants the water flows through 40 miles of conduit consisting of concrete bench and elevated flumes, inverted steel siphons, concrete lined and unlined tunnels and steel penstocks. The flow is handled by automatic and remote controls. One power plant, West Point, is completely automatic.

The Bear River Project means more than just another 40,000 HP. The water stored in Bear River Reservoir will fall through a "one mile drop" to produce power in four P.G.& E. power plants. In addition to the power benefits from Bear River storage are those of flood control, irrigation and domestic water supply. Below Electra plant the water flows into East Bay Municipal District's Pardee Reservior which is one of the major sources of Bay Area water supply.

Rockfill Dam

The damsite required two dams each of about 1200' crest length and heights of 230 and 140 feet (Fig. 2). Geologically there was no problem as massive granite was observed everywhere. The main dam site was clean sound bedrock and required essentially no clearing or excavation. The wing dam site required clearing and some 10 to 40 feet of excavation. The only aggregate located was a river deposit in the reservoir site one mile from the dam, enough for facing rock fill dams but very inadequate for concrete dams. Studies of concrete arch and gravity, slab and buttress and rockfill designs were made. The rock fill dam with concrete face slab (Fig. 3). was selected as the most economic and appropriate structure.

There are 14 rock fill dams of ages from 20 to 80 years in P.G. & E.'s system, some of which date back to the hydraulic gold miners dams constructed in the 1870's. Experience has shown them to be both satisfactory and economic in operation and maintenance as well as in first cost. Salt Springs dam, now 20 years old, has proven very successful. *Concrete face slab cracks have occurred but the annual cost of face repairs has been essentially negligible, they having been required only once in the last seven years. Leakage has been less than required minimum releases and has varied from 5 to 20 cfs. For full reservoir in 1951 the peak leakage was 8 cfs. The design of the 230 ft. high Bear River Dam is largely based on experience with Salt Springs Dam and to a lesser extent on other dams. Discussion will be confined to the main dam, the smaller wing dam presenting no design problems and subject to nominal settlement.

The basic features of the main dam are indicated on Fig. 3. Essentially the dam consists of dumped rock with an upstream layer of placed rock and a reinforced concrete slab face. The rock fill is placed by simply dumping from high lifts (50 to 100 ft.), three times as much water as rock by volume being applied at time and place of dumping rock. The slopes of 1.3:1 upstream and 1.4:1 downstream are conveniently obtained in high lifts since the

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* Handbook of Applied Hydraulics, C. V. Davis.

natural slope is about 1.3:1. The cost of dumped rock is a minimum since it consists essentially of quarrying and hauling costs. The placed rock acts as an equalizer in smoothing out and resisting the settlement of the dam and transmits the appreciable water load from the thin concrete face to the dumped rock fill. The reinforced concrete face slab is intentionally thin and reinforced in the center so that it functions as a flexible watertight membrane.

The design problem of obtaining a watertight membrance with relatively rigid material is appreciated when it is considered that the central portion of the dam will move some three feet downstream while the cut off will remain fixed. Fig. 4 shows a developed plan of the face with joints and predicted contours of settlement. The shape of the face is moderately curved upstream in plan and concave in a line down the face. The slight arching in plan causes a closing of vertical joints due to downstream deflection. From the standpoint of watertightness the closing of joints and compressive stresses are desirable, and further the shortening tends to tighten the placed rock rather than loosen it. The resultant settlement is slightly down from a line normal to the face and produces an appreciable thrust down the face. The concavity in a line down the face is considered desirable as it would tend to prevent buckling. Settlement during construction and particularly under water load insures a concave profile.

Several changes in the face of Bear River Dam, as compared to Salt Springs Dam, have been made which at little cost should tend to reduce maintenance and leakage. Bear River reservoir is 2000 ft. higher in elevation and winter maintenance should be kept to a minimum. Also, the water is "1900 ft." more valuable and any leakage in excess of the required release is serious. The changes, based on Salt Springs experience, in estimated order of importance, are:

1. The sloping joint (Fig. 4) along the abutments and 15 ft. out from the key wall joint is intended to act as a hinge in permitting the face to deflect normal to itself under water pressure. At Salt Springs there were no such joints and cracks developed parallel to the key wall and centered about 15 ft. out.
2. The upper 180 ft. of the five central vertical joints are open 2 in. instead of 1 in. as are the other vertical joints. At Salt Springs they were all constructed open 1 in. and three of the central joints closed beyond the point of crushing failure for a distance of 70 to 120 ft. down from the crest.
3. Horizontal joints consist of a "U" shaped copper seal and 1/2 in. redwood filler. This "soft joint" is anticipated to relieve the compressive stresses in the slab which, as noted above, are great and caused some crushing failures in the face of Salt Springs. Salt Springs horizontal joints were cold joints.
4. Two additional horizontal joints were added through the central area of the face. The curvature of the line of deflection up the face and compression induced by settlement are greatest about half way up the dam. These additional "soft joints" will increase flexibility and relieve compression forces.

The spillway is an open crest double side channel spillway located on a granite knoll between the two dams (Fig. 4). The spillway chute is in granite and unlined. A small radial gate is provided for the sluicing of logs and debris.

Temporary Diversion

A 50 ft. waterfall at the downstream toe of dam made a very economic and simplified temporary diversion and permanent outlet possible (Fig. 3). A 12 ft. high earth cofferdam diverted 3500 cfs through a 10 x 12 ft. unlined tunnel and no downstream cofferdam was required. The tunnel portal grade is set at a maximum flood level to accommodate the valve house of the permanent outlet works. Both driving convenience and maximum cover were obtained by establishing minimum grade from downstream portal to the bottom of a 60^{o} rise. The 60^{o} rise saves length and improves hydraulics over a vertical shaft and is still "self mucking."

Permanent Outlet Works

The temporary diversion tunnel is the permanent dam outlet (Fig. 3). This 1800 cfs capacity outlet works serves several purposes; occasional bypass of up to 600 cfs to Tiger Creek Power House, emergency unwatering and rapid unwatering for face inspection or maintenance.

The 12 ft. diameter outlet tower is capped by a square galvanized steel trash rack with 4 in. clear bar spacing for protection of valves and personnel. The 30 ft. high circular tower is a little higher than necessary for protection of the trash rack against slide debris and waterlogged material in order to permit inspection and maintenance of the complete outlet works. Opening the 3 ft. square slide gate will drain all water below top of concrete tower in approximately 12 hours and permit inspection of the lowest portion of the face of the dam. Closing the gate then permits access to the tunnel while reservoir inflow is stored below the top of the tower. A 6 ft. square rail trash rack with 9 in. spacing protects the 36 in. gate.

The outlet tunnel is routed around the abutment (Fig. 2) for maximum rock cover and being in sound granite remains unlined. Downstream from a well grouted concrete plug is a 72 in. shut-off butterfly valve, a 60 in. pipe and a 60 in. free discharge butterfly valve. No electric power is available at this remote dam and both valves are water motor powered, the 72 in. valve by oil pump and oil pressure cylinder and the 60 inch valve by handwheel or direct geared water motor drive. Free discharge butterfly valves have operated satisfactorily in a number of P. G. & E. installations. Provision for seat adjustment is always provided so that adjustments can be made either outside the valve or downstream in order that any leakage may be stopped without unwatering upstream.

Power Tunnel Intake

A 1200 ft. long open cut leads to the intake which is a 10 ft. square galvanized steel trash rack supported on a 10 ft. high concrete base. The front removable panels extend down to the invert and permits vehicle and equipment access to the tunnel in the event maintenance is required. The clear spacing of bars is 1-1/8 in., which is about 1/5 the turbine jet diameter, and the structure is designed for 15 ft. differential pressure in either direction. An average velocity of 1/3 f.p.s. results from the design flow of 200 cfs through the 600 sq. ft. of trash rack area. The design for 15 ft. of internal pressure is to allow for an upstream flow of 600 cfs from Cold Creek Diversion (Fig. 6). This occasional upstream flow should be an ideal trash rack cleaning feature.

The layout of submerged trash rack with Broome gate in shaft at edge of reservoir was adopted as both the lowest cost and best operating arrangement. The reservoir being at 6,000 ft. elevation is in an area subject to icing and 10 to 15 ft. of snow. No electric power being available the Broome gate lowers without power and is raised by a gasoline engine operated hoist. The gate is normally in the dry at inspection deck level of the hoist house.

Power Tunnel

This tunnel section (Fig. 5) is of particular interest because it is about the minimum size for a long tunnel. The bids called for the 8 x 9 ft. section or any alternate size or shape of equal or better hydraulic properties. The successful bidder, Walsh Construction Company, proposed the 8'- 6" x 11' section shown. The "Gothic" arch was to accommodate the air duct and the additional width to give minimum clearance to reasonable sized tunnel driving equipment. The average measured section was closer to 10' x 11' having 14% overbreak. All the tunnel is unlined.

The value of Mannings "n" for an unlined tunnel is an interesting and indeterminate subject. There are two values; one before construction and on the design area, and the other after construction and on the average measured area. Because of the very appreciable economy in unlined tunnels (where geology permits) measurement and study of the head losses is an important part of the efficiency tests. On different tunnels overbreak has been measured to vary between 6 and 28% and value of "n" on measured area between .032 and .039. As a general rule the head loss should not exceed that for n = .035 on design area. This estimate can be improved with knowledge of the tunnel size and nature of rock formation.

Cold Creek Diversion and Surge Shaft

In a dry year 35% of Bear River Reservoir will be filled by Cold Creek flow, the diversion having a capacity of 800 cfs - 600 going upstream to Bear River reservoir and 200 to the powerhouse. This diversion is thus an essential part of the Project and by making the rise 60^{o} to the horizontal and of 10 ft. square cross section a surge tank was obtained at moderate additional cost. Its use as a surge tank is possible since the shaft is only 1,200 ft. from the beginning of penstock. The limiting condition of design is that the draw down from load on in 1 min. (reservoir at minimum operating level) will not lower the hydraulic grade line to a point less than 10 feet above top of penstock at the portal. The crest of diversion dam is 40 ft. above maximum reservoir water surface in order to divert essentially all Cold Creek flow. This requirement governed approximate location of diversion dam and tunnel alignment and automatically provided an adequate surge chamber for load rejection.

This diversion is in an isolated location. It will be unattended and, in fact, inaccessible some four months of the year. It is a requirement that the diversion function with minimum maintenance. The diversion consists of a concrete ogee section dam, a trash rack structure and manually operated sluice and shut-off gates (Fig. 6). In order that ice and floating debris will not clog the trash rack, it is set horizontal and five feet below the minimum water surface. The water flows under the surface ice and debris, down through the horizontal trash rack and up and over a weir into the "surge" shaft. Additional rack area is provided vertically to better provide

for maximum flow but is expected to become largely plugged. A 15-foot deep pool provides storage for sand and gravel. A 36-inch sluice gate protected by a 6-inch clear spacing grizzly permits unwatering for trash rack cleaning and some sluicing of gravels. A 3-foot wide flash board opening, normally closed, adjacent to the vertical trash rack permits clearance of floating debris from the vertical racks with minimum loss of water.

Rock Trap and Sluice

The tunnel is unlined and requires a rock trap and sluice (Fig. 7). An adequate rock trap is particularly important because of the high head and small nozzles. The use of the term rock trap is conservative as it is hoped that the trap will remove all the rock, most of the gravel and some sand. The matter of paved tunnel invert, because of the small nozzles and high head, was studied and it could not be justified. The unlined tunnel with paved invert would still require a trap but one of lower cost. The saving in tunnel cost in accepting an uncleaned and unlined invert was much greater than the cost of the rock trap, let alone the incremental cost. Since the bottom of an unlined tunnel is the smoothest part of the section, the improvement in hydraulics by invert lining is not very great. Turbine maintenance is another factor and the Company's operating experience substantiates the economics of fully unlined tunnels.

The rock trap is located at a point where depth of cover equals maximum static head (200 ft.). A concrete backfilled steel liner connects the rock trap to the penstock. The planned location of rock trap and length of water tight tunnel section was necessarily arbitrary and was reviewed after the tunnel was driven. It was observed that during rains and snowmelt there was "rain" inside the tunnel up to about 50 ft. from the proposed rock trap. This, and inspection of the rock, confirmed the original location.

The trap (Fig. 7) is deep, has steep side walls and concrete invert to insure that sluicing will remove any accumulation of material. At the bottom a grizzly is provided so that nothing enters the 36 in. sluice pipe that will not pass the 36 in. butterfly. To obtain a deep trap required a 300 ft. long separate small tunnel and the sluice pipe is concrete backfilled to the portal. The 36 in. butterfly sluice valve is backed up by a 36 in. shut-off gate valve. P.G.&E. has had trouble in older sluicing installations with small rocks jamming in gate valve guide grooves and butterfly valves (having no grooves) have been used very successfully on recent installations. The butterfly seal is flush with the inside of pipe and the seat is readily adjustable to eliminate leakage without upstream unwatering. The valve is hydraulic operated, the oil pressure being provided by the same pressure set that supplies the 72" penstock butterfly valve. Hydraulic operation has two desirable features for sluicing; one is the rapid water saving operation, and the other is that the convenience of operation does not make operating personnel reluctant to sluice.

From the sluice valve house the water is conducted in a short discharge pipe to a rock gully that dumps the flow into Salt Springs reservoir (Fig. 1). Not only is sluice water thus salvaged but the sluice may be used as a 200 cfs by-pass while Bear River Unit is running. This feature provides power and storage benefits under certain runoff conditions.

Penstock

Bear River penstock will be the Company's first welded steel penstock for a head greater than 1400 ft. The lower ends of the penstocks for P.G.& E's existing higher head plants, Balch on Kings River (2336 ft.) and Bucks on Feather River (2562 ft.), are of banded pipe made in Poland.

The penstock is 4700 ft. long and varies in diameter from 66 in. to 46 in. and in thickness from 3/8 in. to 1-3/4 in. A 72 in. butterfly valve at beginning of penstock acts as both a shut off and emergency valve. The valve will close automatically in the event of a penstock break. The penstock is supported on concrete piers at 40 ft. centers, piers being at quarter points of the 40 ft. penstock sections. Steel rocker piers are used for heights greater than 5 ft. The brief review of design, fabrication, erection and painting which follows is essentially P.G.&E.'s current practice.

Since 1946, P.G.&E. has used high strength silicon alloy A-212B steel. Its minimum elastic limit of 35,000 psi effects a saving of some 23% in tonnage over the customary mild steels. A working stress of 15,000 psi for maximum static head plus 10% waterhammer was used, giving a safety factor of two and one third on the elastic limit. The rate of nozzle and turbine valve closure is adjusted to keep normal operating pressure rise less than 10%. To obtain maximum rate of needle closure for best plant operation, a dash pot action is obtained near the closing of the needle stroke where rate of shutting off the water is greatest. The servomotor piston closes off one orifice at about 0.1 needle opening. Since the spherical valve shuts off essentially all the flow in the last 20^{o} of rotation, a dash pot for reduced rate of closure at end of closing is again provided to obtain maximum rate of closing without excessive water hammer or surge.

The practice of field riveting the shop welded, x-rayed and stress relieved penstock sections is considered to insure a ductile, high strength shell free of internal stresses. Shop fabrication is in 40 ft. lengths with all welds made by the automatic submerged arc process. To avoid field welding on the A212B steel, a riveted joint (Fig. 8) was developed that would minimize field work. After welding and fitting up all butt welds are x-rayed and any required chipping and re-welding is done. On the recently completed Rock Creek job, 11% of welds were chipped and re-welded. The pipe sections are then carefully handled until stress relieved in accordance with the API code. All shop work is continuously inspected.

Pipe sections are tightly fitted in the shop to a tolerance of less than 1/32 in. on the diameter. To make field entry easy, four small brackets are shop welded to each end of each section. After the pipe is rolled in place, it is roughly wedged and jacked to approximate line, and four 20-ton centerhole hydraulic jacks are used to pull the sections together. The shop sub-drilled holes are reamed, then cone-head rivets are put in from the outside and an oval countersunk head is driven on the inside of the penstock. The use of this joint on six recent penstock jobs has been very satisfactory in speed of erection and in low erection costs.

The pipe receives a shop spun coat of coal tar enamel on the inside and a coat of white wash on the outside. The hydraulic advantage of the smooth spun coating over hand daubing is desirable as well as obtaining a better and cheaper coating. Little blistering or cracking in shipping has been experienced. The last operation before filling penstock is a complete spark test of both hand daubed field joints and shop spun coating. The white wash, in reducing temperature of the steel, minimizes tendency to blister and

reduces movement between expansion joints before penstock is filled. Hot coal tar enamel interior penstock coating has been P.G.&E. practice since 1930 and inspection of penstocks has shown all penstocks to be in practically perfect condition.

For the exterior exposed coating the pipe is sand blasted to bright metal and two coats of red lead primer and one of aluminum paint are applied. The exterior protective coating is applied a few years after the penstock is in operation during a period of good weather and at the convenience of painting crews. The peeling of mill scale during this period is desirable since it makes sand blasting easier.

Power House and Turbine

When Salt Springs power house was constructed in 1932, space was allowed for the future installation of Bear River Unit. The 255 ft. Salt Springs Frances turbine and the 2110 ft. head impulse turbine both discharge directly into a flume which carries the water to the 1219 ft. head Tiger Creek power house.

Study of the particular requirements of the Bear River installation and of the most recent impulse wheel developments resulted in P.G.& E. Turbine Specifications calling for a vertical shaft multiple nozzle impulse turbine. The primary considerations leading to this choice were:

1. Just 20 ft. beyond centerline of unit, the water enters the side of Tiger Creek flume and immediately downstream is a long side channel spillway that operates with about 0.1 ft. freeboard. The velocity and turbulence in the tailrace are much lower for the vertical shaft multiple nozzle turbine for normal operation. The self contained energy dissipator is a desirable feature for load rejection.
2. The space requirements in the existing power house were well adapted to the vertical shaft machine.
3. Having three nozzles the failure or complete plugging of a nozzle will only produce 1/3 the water hammer of the failure of a single nozzle machine. This is valuable in security of the penstock.
4. The higher efficiency as evidenced in model and full scale tests. This may amount to 2 and possibly 3 percent.
5. Assured higher efficiency at low loads due to operation on one or two nozzles. This efficiency benefit is on the order of 5% and Bear River Unit operation will require partial load operation.

Operation and Tests

Operation began in November 1952 and with it began operating experience. The tailrace, both for normal operation and rejection, was even quieter than hoped for and required no alterations to the flume. Reliability of Cold Creek Diversion, amount of maintenance of buckets and other experience data will require time to obtain.

Efficiency tests will soon be made by the Allen Salt Velocity method. These tests will include tunnel and penstock head loss measurements and surge chamber tests.

Observations on settlement points on the two rock fill dams and of leakage will soon contribute to the knowledge of rock fill dams. Since the primary settlement seems to occur approximately in equal increments with the first two reservoir fillings, the measurements in winter of 1953 will be of value.

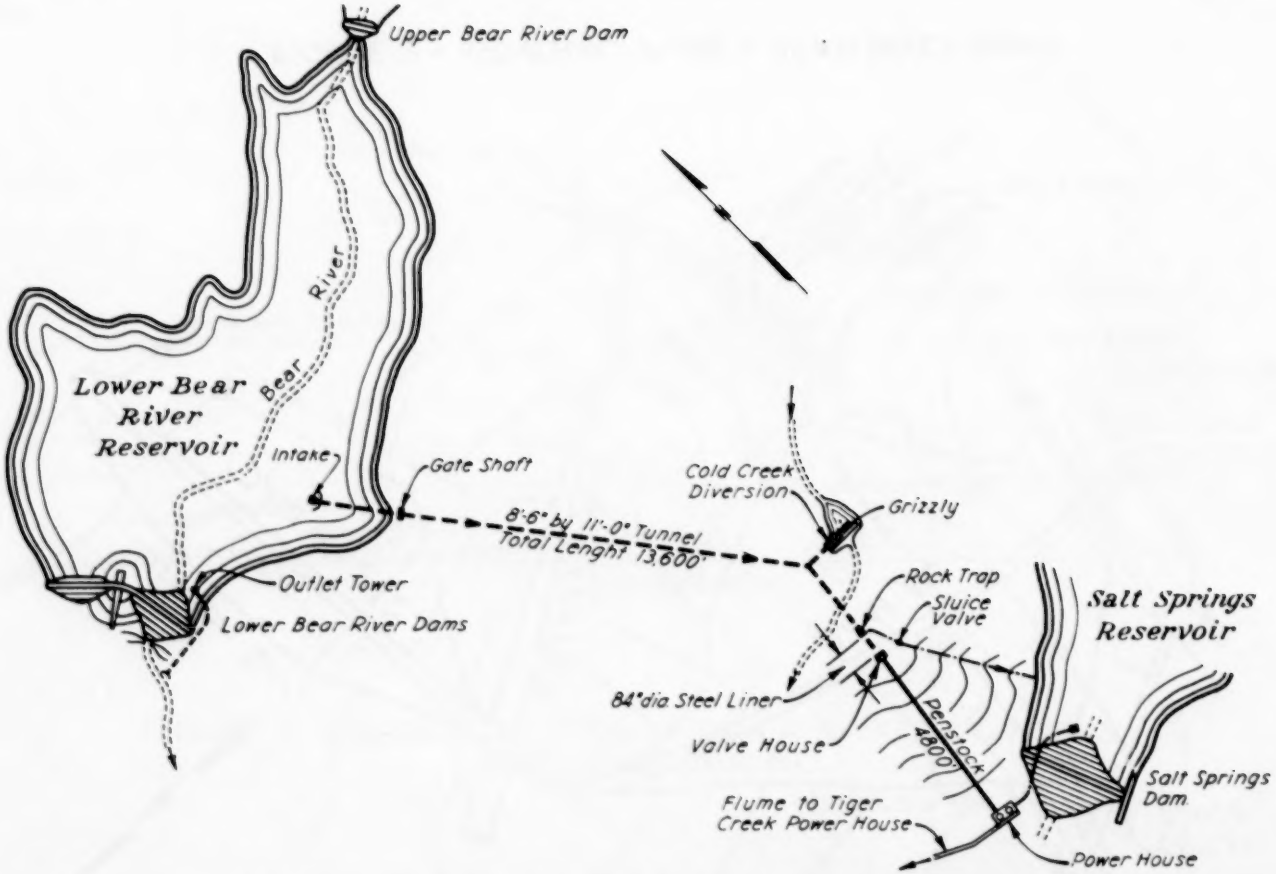

FIGURE 1 - GENERAL PLAN - BEAR RIVER PROJECT

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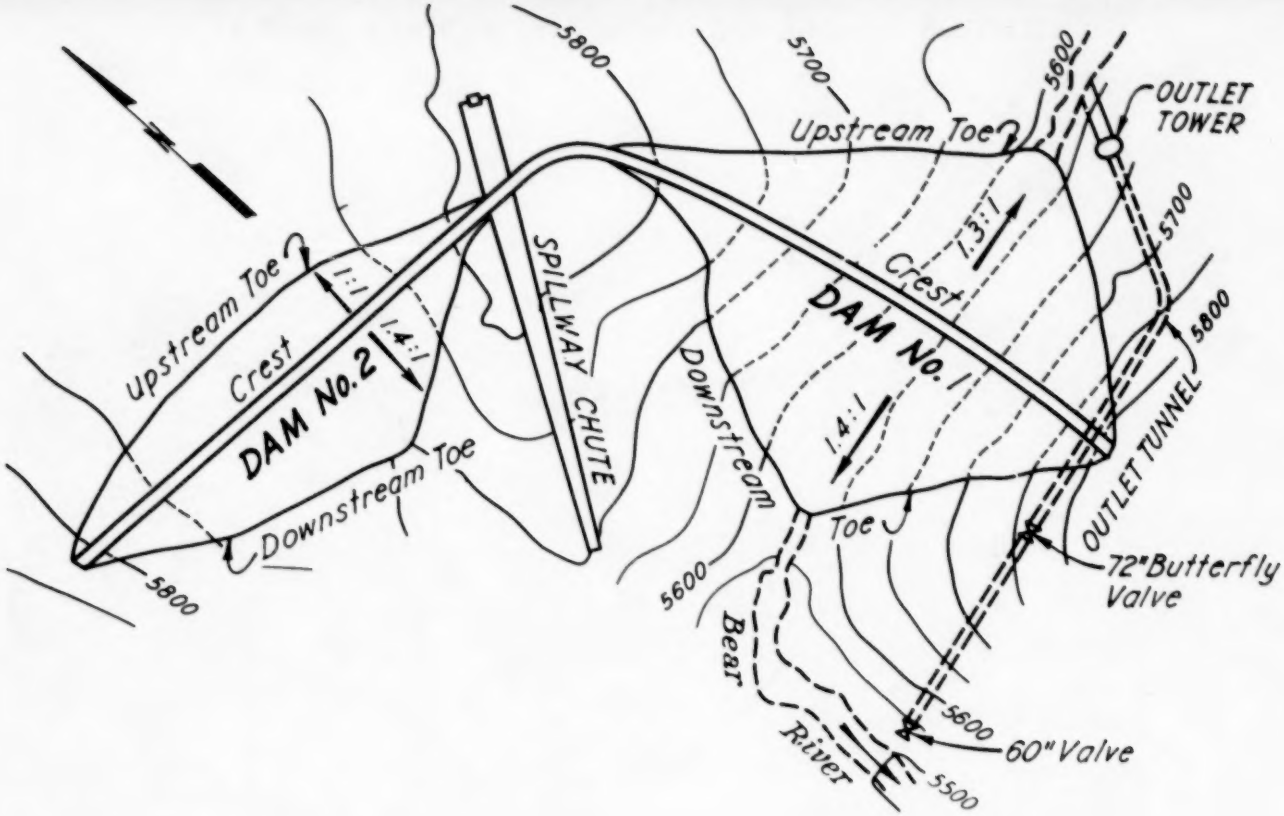

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FIGURE 2 - GENERAL PLAN - BEAR RIVER DAMS

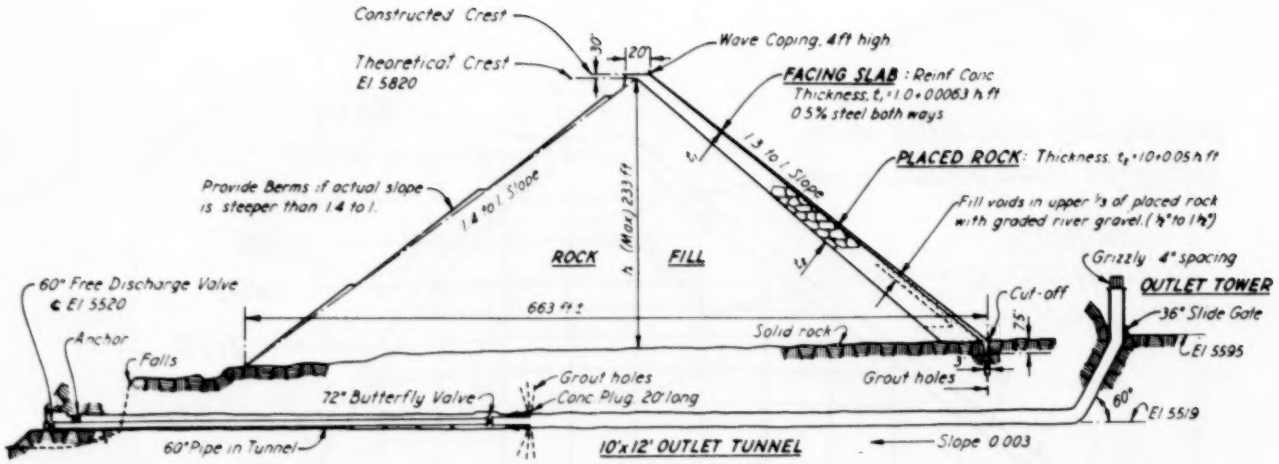

FIGURE 3 – MAXIMUM SECTION OF DAM AND OUTLET WORKS
BEAR RIVER DAM NO. 1

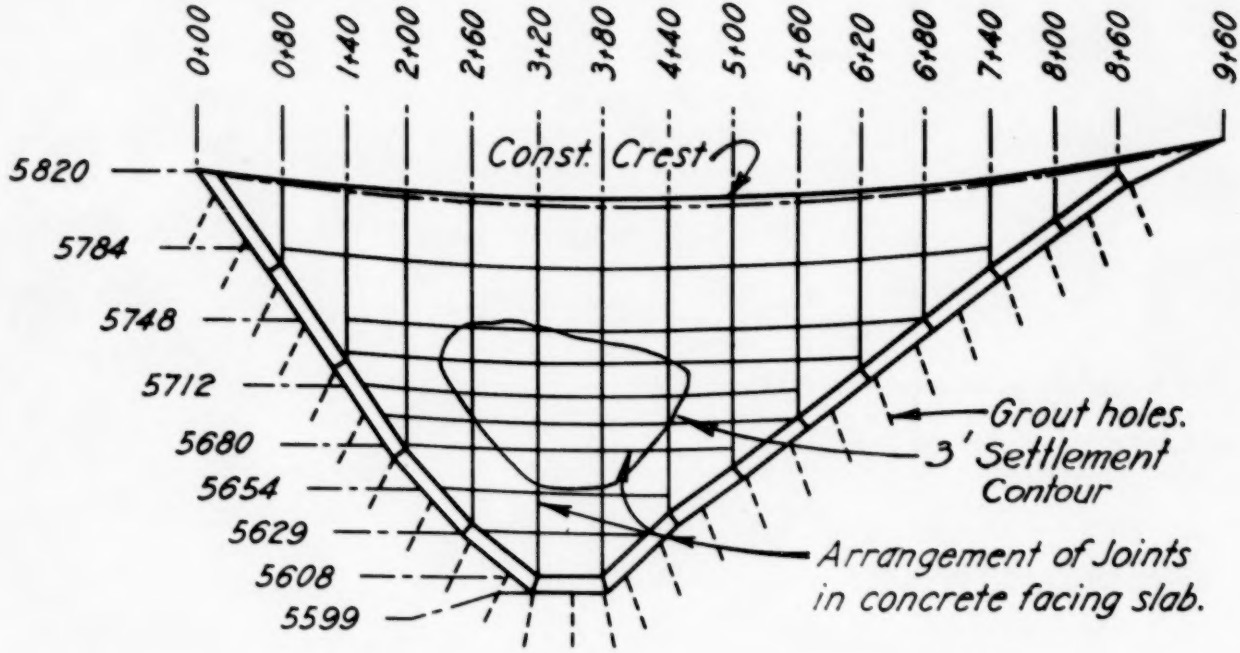

FIGURE 4 - PLAN OF DEVELOPED UPSTREAM FACE
BEAR RIVER DAM NO. 1

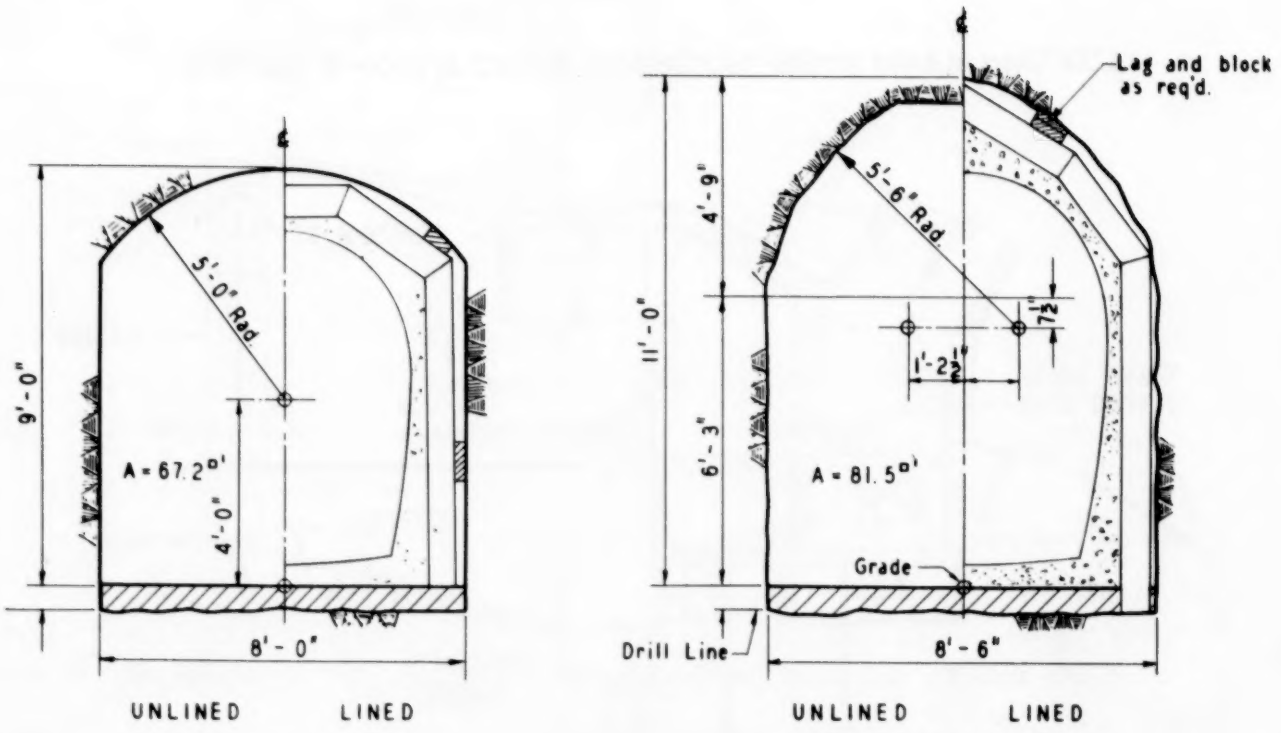

FIGURE 5 – BEAR RIVER TUNNEL SECTION

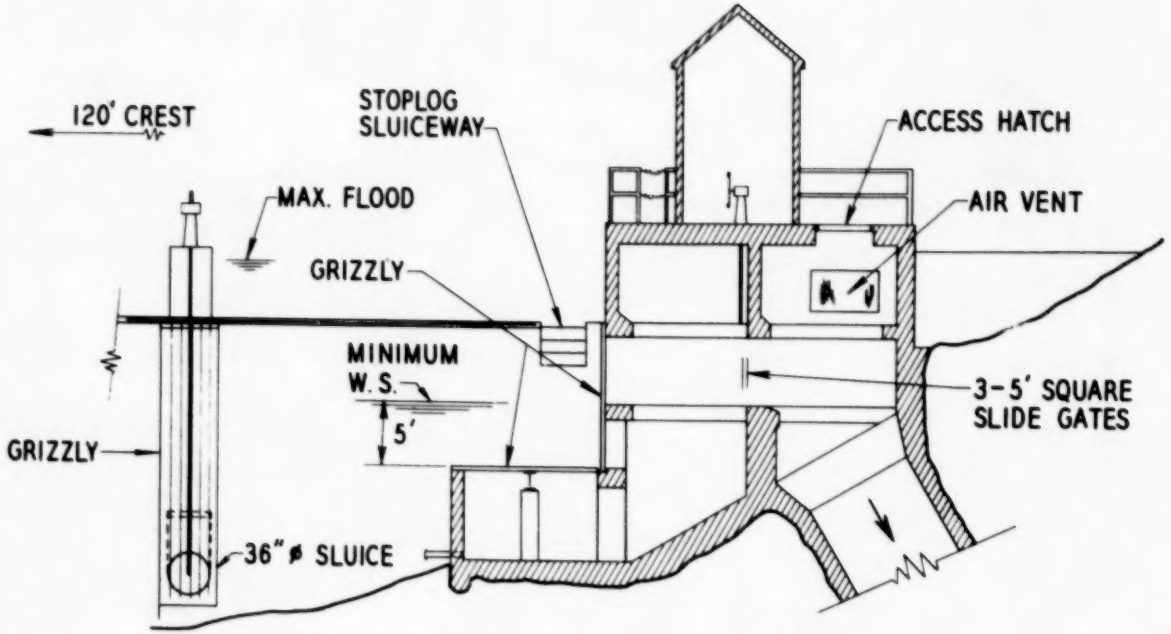

FIGURE 6 - COLD CREEK DIVERSION - BEAR RIVER PROJECT

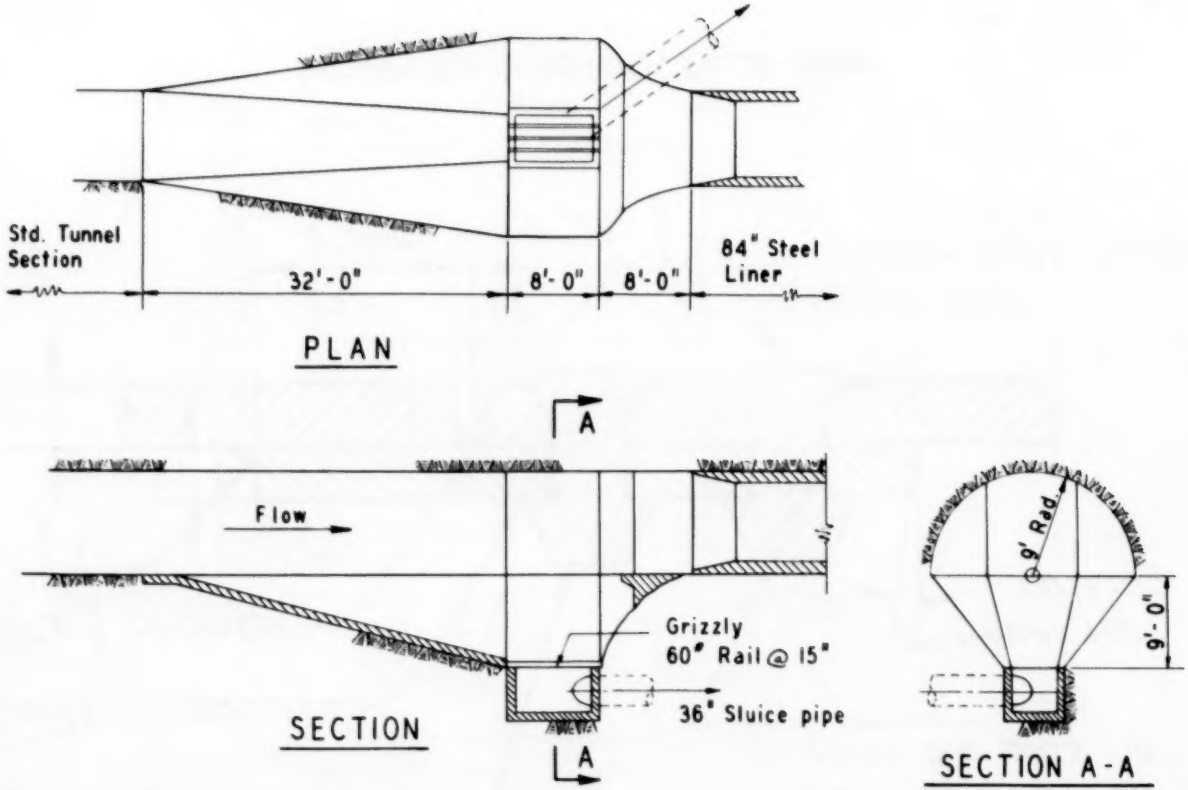

FIGURE 7- ROCK TRAP-BEAR RIVER TUNNEL

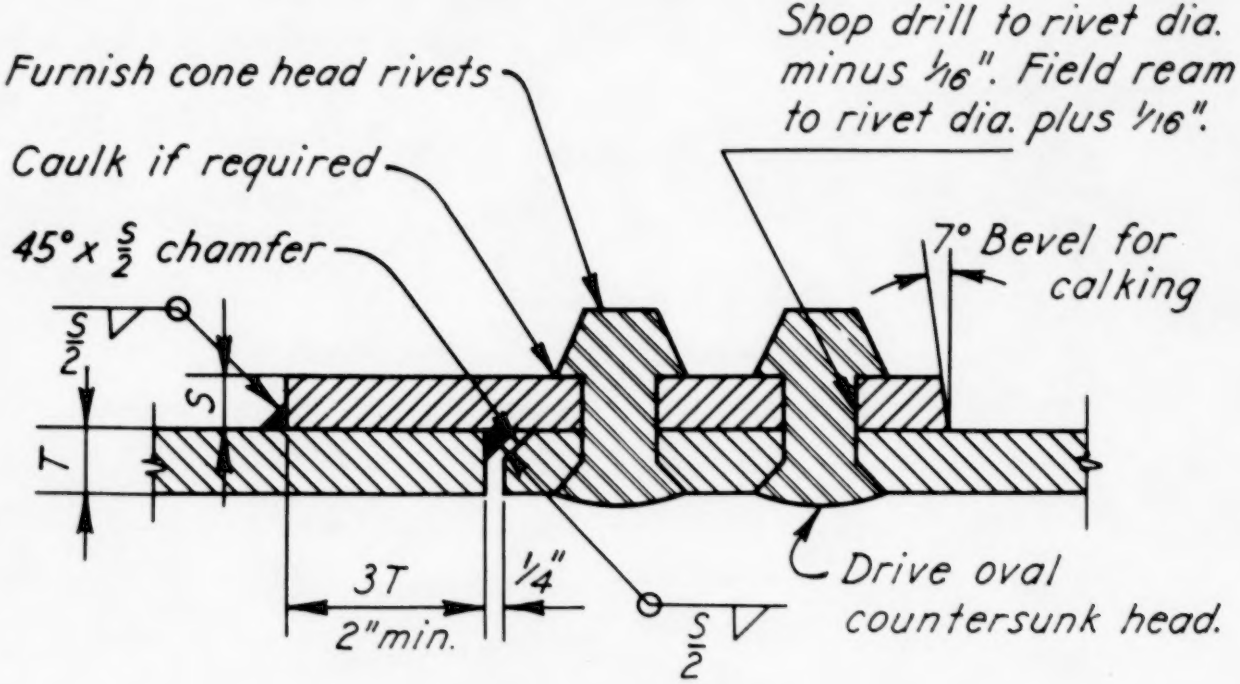

FIGURE 8 – PENSTOCK FIELD JOINT

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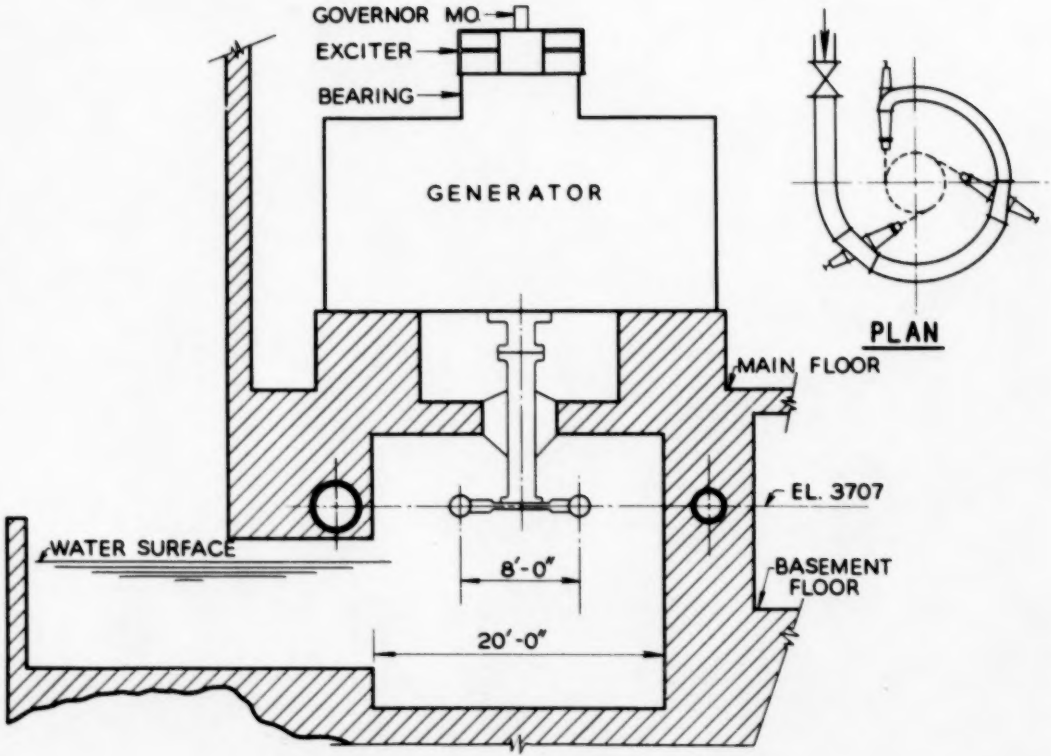

FIGURE 9 – BEAR RIVER UNIT – SALT SPRINGS P. H. – 40,000 H. P.

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GLENN W. HOLCOMB
OLIVER W. HARTWELL
ERNEST W. CARLTON
DON M. CORBETT